CHAPTER 130

The dynamic response of shingle beaches to random waves

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Abstract

An extensive laboratory investigation into the behaviour of shingle beaches has been undertaken using a large random wave flume. The study utilised a lightweight material scaled to reproduce the correct permeability of the beach, and the correct threshold and relative magnitude of the onshore/offshore movement. Results are presented describing both the wave reflection characteristics of the beach and the probabilistic distribution of wave run-up crests on the foreshore. Where possible the laboratory results are validated against field data.

1 Introduction

Although relatively scarce on a worldwide basis shingle, or gravel, beaches (D_{50} = 10 - 60mm) are a common feature around the UK coastline. However it is only in recent years that their considerable merits as coast protection structures have been fully recognised, and only over the last few years that this belated recognition has been transformed into a more widespread engineering application. This application is still restricted, however, not only by a paucity of information, regarding shingle beach processes under wave action, but also by a general lack of understanding as to how information which is currently available should be applied to a particular problem. In response to this situation an extensive series of model tests have been undertaken in the UK in recent years using both regular (Powell, 1986) and random waves. This paper summarises the procedures and results of the random wave investigation with particular emphasis on the wave reflection coefficients and wave run-up distributions.

2 Scaling Criteria for Model Beach Sediment

In order that a mobile bed physical model may accurately simulate natural beach processes it is necessary to ensure that the sediment used in the model is representative of that occurring in nature. For shingle beaches the model sediment should ideally satisfy three main criteria:

a) The permeability of the shingle beach should be correctly reproduced.

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b) The relative magnitudes of the onshore and offshore motion should be constant.

c) The threshold of motion should be correctly scaled.

The first of these criteria basically governs the beach slope, the second determines whether the beach will erode or accrete under given wave conditions, and the third determines the wave velocity at which sediment motion will begin.

Yalin (1963) published a paper describing a method for modelling shingle beaches with the correct permeability and drag forces. For the permeability he stated that in an undistorted model the percolation slope must be identical to that of the prototype beach

$$\text{i.e.} \quad \lambda_V^2 \, \lambda_K \, / \, \lambda_D = 1 \tag{1}$$

where λ is the model scale
V is the flow velocity through the voids
K is permeability, a function of the voids Reynolds number.... $V D_{10} / \nu$
D is sediment diameter
and ν is kinematic viscosity

Assuming that the model is operated according to Froude's Law this equation can be re-written as

$$\lambda_D = \lambda K_p \, / \, K \, (Re_p \, / \, \lambda^{\frac{1}{2}} \, \lambda_D) \tag{2}$$

where the subscript p refers to prototype values. Provided that K_p, Re_p and the form of the function K ( ) are known this equation can then be solved by successive approximation to define the particle size for the model sediment, for a given model scale.

The correct reproduction of the relative magnitudes of the onshore/offshore sediment motion requires the similitude of the dimensionless fall velocity parameter, $H_b/\omega T$, (see Shore Protection Manual, 1984).

$$\text{i.e.} \quad \lambda H_b \, / \, \lambda_\omega \, \lambda_T = 1 \tag{3}$$

where H_b is the breaking wave height
ω is the sediment fall velocity
and T is the wave period

However the settling velocity for a sphere may be approximated by

$$\omega = \left(1.33 \, gD \, (\rho_s - \rho_f) \, / \, C_D \, \rho_f\right)^{\frac{1}{2}} \tag{4}$$

where ρ_s and ρ_f are specific gravities of the sediment and fluid respectively, and C_D is the drag coefficient for the settling particles (Rouse, 1950).

For a Froudian model therefore, assuming that the beach slope is correctly reproduced, equations 3 and 4 may be combined to yield an expression for the specific gravity of the model sediment:

$$\lambda\rho_s' = \lambda\ \lambda_{C_D} / \lambda_D \tag{5}$$

where $\rho_s' = (\rho_s - \rho_f) / \rho_f$ (6)

and λ_{C_D} is given as a non-linear function of the sediment particle Reynolds number by

$$\lambda_{C_D} = C_{D_p} / C_D\ (Rep / \lambda^{\frac{1}{2}}\ \lambda_D) \tag{7}$$

Again, if C_{D_p} and Re_p are known, and λ_D has also been determined (i.e. equation 2), then equations 5, 6 and 7 can be solved for ρ_s, the specific gravity of the model sediment.

A second expression for the specific gravity of the model sediment is obtained through consideration of the threshold of motion of the sediment particles. For oscillating flow Komar and Miller (1973) proposed that for sediment sizes greater than 0.5mm, which is usually the case for shingle beach models, the threshold of movement would be defined by the expression,

$$U_m^2 / \rho_s'\ gD = 0.46\ \pi.\ (d_o/D)^{0.25} \tag{8}$$

Where U_m is the peak value of the near bed orbital velocity at the threshold of motion and d_o is the near-bed orbital diameter. Re-working equation 8 and assuming a Froudian model yields the expression,

$$\lambda\rho_s' = \left(\frac{\lambda}{\lambda_D}\right)^{3/4} \tag{9}$$

It should be noted, however, that equation 7 was originally developed for predicting the threshold of movement of fully submerged gravel on a horizontal sea-bed subject to non-breaking waves. This is a very different situation to an emergent, sloping shingle beach subject to breaking waves, so the accuracy of the equation cannot be assured.

Generally equations 5 and 9 give conflicting requirements for the specific gravity of the model sediment, ρ_s, and one or other of the equations usually needs to be relaxed. This complication in the modelling of beach sediments is further compounded by the fact that there is only a very limited range of specific gravities amongst the readily available materials. Frequently, therefore, the selection of the model sediment is governed as much by availability as by theoretical considerations.

For the present study the scaling requirements have been plotted in graphical format in Figure 1. These may be satisfactorily met at a scale of 1:17 by the use of crushed anthracite (coal) which has a specific gravity of 1:39, and is readily available in a range of gradings. At the selected scale the specific gravity requirements for the model sediment, based on the threshold and direction of motion criteria are respectively 1.45 and 1.35, which are acceptably close to the value for anthracite.

3 Laboratory procedures and test programme

During the course of the study four different sizes, and two different gradings, of beach material (selected to cover the range of shingle sizes commonly found around the UK coast) were used to build the model beaches. The beaches themselves were always constructed at a 1:7 slope but up to 5 different depths of beach material were tested. Each beach was subjected to up to 29 wave spectra of the JONSWAP type covering the following range of conditions:

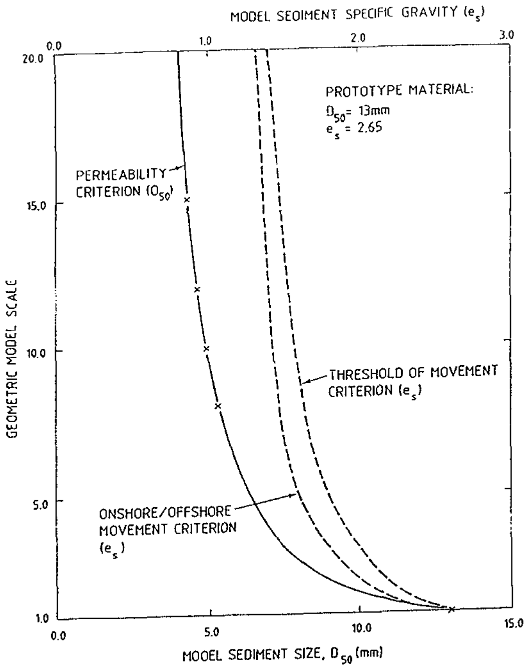

Fig 1 Sediment scaling criteria

$0.05m \leq H_s \leq 0.175m$

$0.8s \leq T_z \leq 2.7s$

$0.005 \leq H_s/L_z \leq 0.06$

The depth of water at the toe of the model was kept constant at 0.8m throughout the course of the study.

Amongst the measurements recorded during the investigation were the wave reflection coefficients for the model beaches and the wave run-up exceedance distributions on the beach face.

Wave reflections were recorded using 3 wave probes located at set positions along the centreline of the flume. The incident and reflected wave spectra were not directly measured but were calculated using an analysis program devised by Gilbert and Thompson (1978) and based on the method of Kajima (1969). The analysis method calculates values of K_r over a wide range of frequencies, but the procedure is only valid over a restricted band related to the probe spacing. For the current study, the use of three wave probes effectively provided three different probe spacings thus allowing a wide range of frequencies to be covered.

Because beaches are constantly adjusting their form in response to the incident wave conditions, it seems likely that the proportions of wave energy reflected or dissipated may also vary as the beach gradually evolves. To test this hypothesis three sets of measurements were made per experimental run, after allowing the beach an initial development period of 500 T_z.

Attempts to record wave run-up distributions in laboratory beach models usually meet with two main problems.

1) The mobility of the beach - which tends to restrict the use of instrumentation on the beach face itself.

2) The presence of edge effects along the side walls of the flume - which can affect visual recordings taken at beach level.

To overcome these problems a simple method was developed for measuring wave run-up distributions along the centre line of the flume. This involved blacking out the half of the flume furthest from the observer whilst lighting the front half from above. The image of an illuminated marker board located outside the flume, and referenced to still water level, was then reflected into the flume and projected on to the centre line boundary, between the light and dark sections of the flume. The marker board was drawn up with twelve numbered bands in such a way that its image appeared correctly orientated. With a little practise an observer was able to record the total number, and hence proportion, of wave run-ups exceeding specified levels against the image of the marker board. Initial proving of the method demonstrated a high degree of repeatability, which appeared to be independent of the observers involved.

4 Wave Run-up distributions

Generally five wave run-up recordings, of $300T_z$ duration, were taken for each test, with each recording being separated by a 200 T_z interval. Prior to any measurements being taken the profile was allowed to evolve naturally for a period of $500T_z$. This was generally long enough for the major profile features to develop and subsequent analysis of the results showed no evidence of any duration dependent trends.

On completion of a test the records were processed to provide, firstly, the cumulative number of wave run-ups exceeding a specified level per record; and then, secondly, the combined exceedance probability of wave run-up, for those levels, for all five sets of data. The resulting probability distribution was then used in all subsequent analysis.

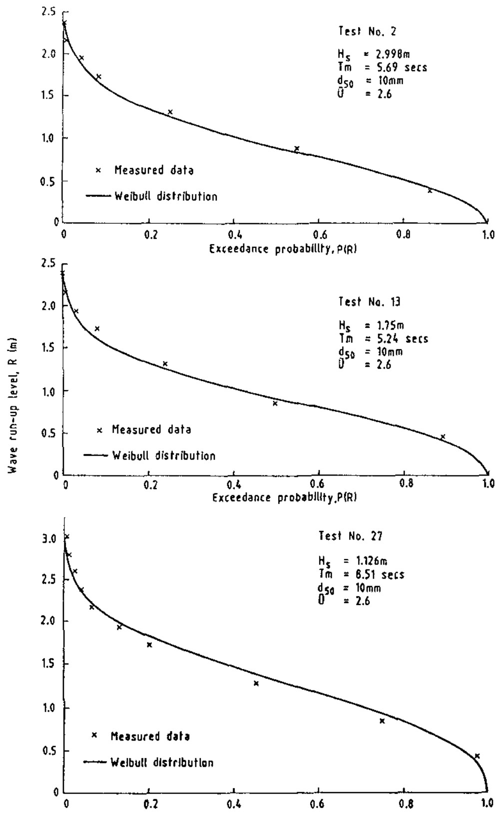

Fig 2 Typical wave run-up exceedance distributions

Typical wave run-up distributions recorded during the model tests are given in Figure 2. These measured distributions have been tested against theoretical Weibull and Rayleigh distributions of the form:

$$\text{Weibull}: \quad P(R) = \exp\left(-B\,(R-C)^{A}\right) \tag{10}$$

$$\text{Rayleigh}: \quad P(R) = A\exp\left(-\frac{BR^{2}}{2}\right) \tag{11}$$

where A and B are curve fitting coefficients
C is a lower limiting value of R
R is a specified level relative to still water level
and P(R) is the probability of a wave run-up exceeding R

In this particular instance the lower limiting value C in the Weibull distribution has been taken as zero, and the run-up has therefore been measured relative to still water level rather than to an arbitrary mean water level which would necessarily include the component due to wave set-up.

Generally the differences between the two distributions are small when compared to the measured data. However the Weibull distribution usually returns slightly better correlation coefficients and hence provides a better fit to the data, particularly over the lower end of the range. The Weibull distribution was therefore taken as providing the best description of wave run-up on the model beaches. This is confirmed by Figure 2 where the theoretical Weibull distribution shows good agreement with the model data.

Detailed analysis of the results suggests that the Weibull coefficient B is a function of both wave height, H_s, and mean sea steepness, H_s/L_z. The precise form of the relationship is given in Figure 3 from which regression analysis yields:

$$B = 0.3\left[H_s^{1.4}\exp\left(-30.0\,H_s/L_z\right)\right]^{-1.6} \tag{12}$$

with a correlation coefficient, r = 0.96.

As may be seen, B is therefore proportional to H_s/L_z but inversely proportional to H_s. Thus for a constant value of A, increasing B (i.e. increasing sea steepness or decreasing wave height) reduces the probability of the wave run-up exceeding a specified level.

In contrast to coefficient B, the values of coefficient A appear to be largely independent of wave climate with a mean value of 2.2 and a standard deviation, $\sigma = 0.22$. Combining this mean value with equations 10 and 12 yields an expression for determining the probable distribution of wave run-ups on a shingle beach, relative to still water level,

$$\text{i.e.} \quad P(R) = \exp\left(-BR^{2.2}\right) \tag{13}$$

where the value of B is given by equation 12.

Note that although equation 13 does not include any allowance for variations in beach sediment size, this was not found to be a serious handicap. Indeed throughout the test series no dependency between wave run-up and the beach material characteristics could be observed.

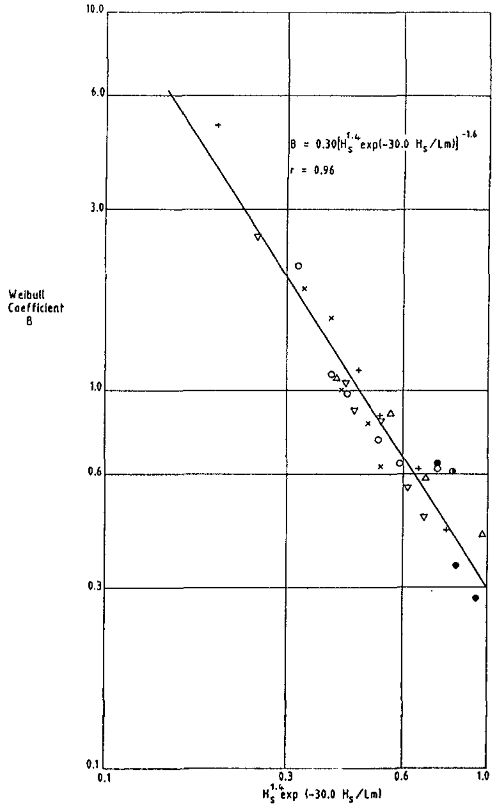

Fig 3 Weibull distribution - coefficient B

The applicability of equation 13 to the test results is confirmed by Figure 4. Here data gathered during field measurement exercises at Chesil Beach and Hurst Castle Spit (on the south coast of the UK) is compared with the predictions of equation 13 for values of P(R) = 0.5 and 0.02. As can be seen the agreement between the predicted and measured values is good.

Observations made during the course of the test programme suggested that, even when fully developed, the beach crest would be overtopped by a small percentage of the wave run-ups. Analysis of the data, based on the assumption that it fitted a Weibull distributions, allowed this percentage to be estimated for each test condition. The resulting exceedance probabilities, calculated for a crest height formed after 3000 waves, suggest that generally less than 3% of the wave run-ups overtop the beach crest, with the mean probability of overtopping $P(R > h_c) = 0.015 \pm 0.011$. No systematic variations, based on wave conditions, are apparent within the results.

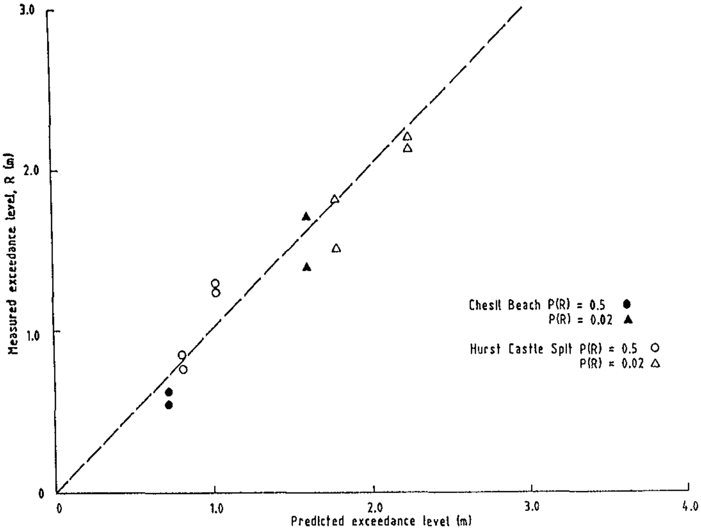

Fig 4 Comparison of measured and predicted run-up exceedance levels

5 Wave reflection coefficients

The effectiveness of a shingle beach in dissipating wave energy is an important measure of its usefulness as a coast protection structure. This is particularly true of beaches used as energy absorbing structures in enclosed waters (i.e. marina's etc) where high levels of reflected energy can have undesirable consequences for small vessels.

During the course of the present study an extensive series of measurements were made of wave reflection for a variety of wave and beach conditions. Generally three sets of reflection measurements were collected for each test, after first allowing a period of $500T_z$ for the main features of the beach to evolve. The reflection measurements were taken between 500-1000, 1500-2000 and $2500\text{-}3000T_z$ from the commencement of the test. On-line analysis of the results produced details of the incident and reflected wave spectra together with values for the reflection coefficients, both for discrete frequency bands within the spectrum and for the wave spectrum as a whole.

Values of the characteristic reflection coefficient K_r (defined as $(Sr/Si)^{\frac{1}{2}}$ where Sr and Si are respectively the reflected and incident energy densities) calculated for each wave spectrum were found to be related to the incident spectral sea steepness, H_s/L_z. This relationship is depicted in Figure 5 for all K_r values obtained. The resulting trend shows that the proportion of wave energy reflected by a shingle beach is reasonably constant, at around 10%, for all values of sea steepness greater than 0.02 (i.e. breaking wave conditions). For sea steepnesses less than 0.02 the effectiveness of the beach in dissipating wave energy reduces rapidly (i.e. higher levels of reflected energy). It is interesting to note that Figure 5 shows the material size, D_{50}, and the effective beach depth, D_B/D_{50}, (where D_B is the vertical depth of beach material) to be of little consequence to the overall K_r trend. This may suggest that wave energy is primarily dissipated in the processes of wave breaking, and overcoming frictional losses in flow over and within the surface layers of the beach. If this is so, flow within the body of the beach can be considered to add little to the overall dissipation of wave energy.

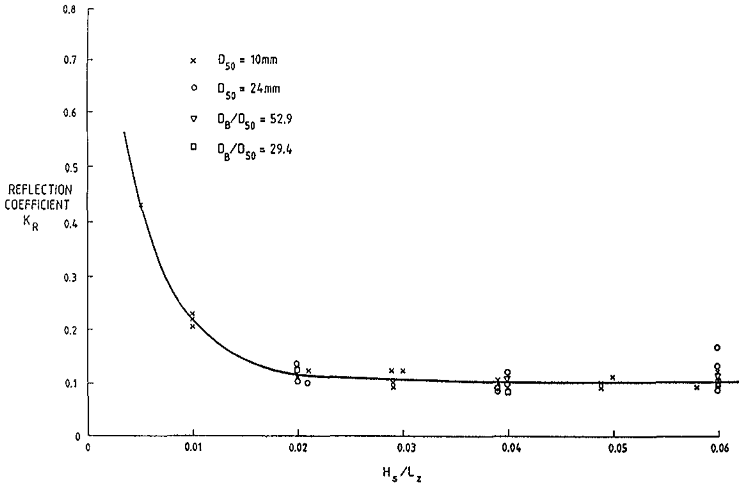

Fig 5 Wave reflection coefficients

6 Conclusions

The wave run-up and wave reflection characteristics of shingle beaches have been investigated using a scaled physical model in a random wave flume. The correct response of the model beach has achieved through the accurate reproduction of both the beach permeability, and the threshold and direction of sediment motion. So far the conclusions arising from the study may be summarised as:

1) The probabilistic distribution of wave run-up crests on a shingle beach is most closely described by a Weibull distribution.

2) The probability of the run-up exceeding any given level on a shingle beach can be adequately determined from equations 12 and 13.

3) Using these equations the proportion of wave run-ups exceeding the wave-formed beach crest is found to be generally less than 2%, regardless of beach material size or the wave conditions.

4) The proportion of normally incident wave energy reflected by shingle beaches is nearly constant, at around 10%, regardless of the size, grading or active depth of the beach material.

5) For wave steepnesses greater than 0.02 the proportion of energy dissipated is also virtually independent of the incident wave conditions.

If should be noted that these results apply only to shingle beaches subject to normally incident wave action. Further research is underway to determine the effects of oblique wave attack on the wave run-up and reflection parameters.

7 Acknowledgements

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8 References

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